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STAGE 1 GEOTECHNICAL STUDIES FOR INTERSTATE 15 RECONSTRUCTION PROJECT, SALT LAKE COUNTY, UTAH

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ABSTRACT

Interstate 15 Reconstruction Project includes rebuilding of 137 bridges; widening the existing three general purpose lane roadway to four general purpose lane with a HOV lane and an auxiliary lane roadway, and other associated work such as converting the existing diamond interchanges to single point urban interchanges (SPUI). The project will be built under a design-build procurement process and is anticipated to be completed in 4 ½ years by October of 2001.

The subsurface soils beneath the corridor consist of lake deposits (Lake Bonneville), namely soft to medium stiff plastic clays, silts and loose to medium dense sands ranging in thickness over 150 meters. The shallow water table is generally 5 to 10 feet below natural grade. The soft clays have low shear strengths and are highly compressible under embankment loads. The Salt Lake segment of the Wasatch fault is approximately 3.5 kilometer to the east of the highway corridor. The structures will have to be designed to meet the seismic criteria and take into account the high liquefaction potential of some of the saturated sand lenses.

Stage 1 efforts included identification of the various subsurface conditions; evaluation of soil parameters; establishing guidelines for field investigations, laboratory testing; analysis; reporting etc. In addition, various project specific studies were carried out for the proposed reconstruction project, details of which are presented in the paper.

KEYWORDS

Surcharge
Wick drains
Settlement
Liquefaction
Seismic Criteria
CPT
Earthquake
Wasatch Fault
Design Spectra
Lake Bonneville

INTRODUCTION

Interstate 15 reconstruction project consists of rebuilding 17 miles of highway extending from 10800 South to 600 North in Salt Lake County, Utah (Figure 1). The general topography within the corridor is relatively flat with the exception of some localized areas of rolling hills. The project site is underlain by geologic units that include bedrock along the valley margins and unconsolidated deposits of alluvial, colluvial and lacustrice origins in the valley. Thicknesses of the unconsolidated deposits range from less than 3 feet on mountain slopes to more than 2,000 feet beneath the Salt Lake valley. The subsurface soils beneath the corridor alternate between deep-water lake deposits and near-shore shallow water deposits of Lake Bonneville. The near-shore deposits are mainly loose to dense sands, silts and gravel in varying percentages. The deep-water deposits consist principally of alternating loose to medium dense silty sand. The top 30 to 100 feet below ground surface consists of soft to medium stiff plastic clay and loose silty sand. This layer is generally underlain by soft to medium stiff clay which in turn overlies medium dense to dense sand and gravel. Depths to groundwater vary, but in general is encountered at shallow depths of about 5 to 10 feet and at greater depths of about 49 feet below the ground surface. Artesian condition is encountered at the north end of the project.

Based on the subsurface conditions at the project site, the following were determined to be critical to the design of the project: a) Embankment settlements; b) Embankment stability; c) Time-rate settlement of embankments; d) Adverse effect of high groundwater; e) Seismic hazards including liquefaction potential and f) Settlement impacts on structures (bridges, walls, adjacent structures, properties).

A geotechnical review committee consisting of Utah Department of Transportation (UDOT) employees namely Ed Keane, Jon Bischoff, Jim Golden, Dave Nazare, Si Sakhai, and Carlos Braceros; Parsons Brinckerhoff (PB) employees namely Thomas Lee, Dan Church and K.N. Gunalan; Bill Gedris with FHWA and PB's subconsultant Dames & Moore's John Wallace was established to direct and oversee the development of geotechnical investigation, analysis and design criteria for the project. This committee oversaw the compilation of the existing data and outlined the scope of work for the Stage 1 investigation. Stage 1 investigation was conducted primarily by Dames & Moore, Inc. as a subconsultant to PB, the Program Manager for the project.

HISTORICAL GEOTECHNICAL DATA

Original construction of I-15 which began in 1959 was done in segments with the final segment being completed in 1969. Geotechnical information for the original design was based on data collected from approximately 300 borings, ranging in depths from 5 meters to 45 meters between 1958 and 1962. Laboratory tests included moisture content, atterberg limits, consolidation, unconfined compressive strength tests and triaxial compression tests. Based on this information time required for 90% and 100% settlement were calculated. Based on the information obtained and using the current state of knowledge, sand drains along with surcharge was determined to be the means to accelerate settlement. Embankments were constructed in stages (in 1 foot or larger lifts). In certain locations the surcharge was allowed to stay on for as long as four years. Field measurements of settlements as recorded, ranged from 0.05 meters to 1.69 meters for fill heights ranging from 5 to 14 meters. Based on the recorded information and observations made by UDOT employees at that time and as indicated during our discussions, it appears that the sand drains installed to accelerate settlement were not very effective.

UDOT considered the available historical data pertinent to the I-15 reconstruction project and with the assistance of PB compiled the information in a report titled "Report on Historical Geotechnical Data". The information contained in the report included data from available historical field and test data reports; design reports for highway fills and structure foundations; data from borings drilled between 1992 and 1995; a summary of laboratory test data; data on existing foundation (shallow and deep foundations), fill surcharge, sand drains and settlement plates; and information on existing groundwater data. This information was used in the preliminary estimates of settlements of proposed embankments (with fill heights 18 feet or more) and the evaluation of associated stability which was included in the General Development Plan of the project. For the study where there was insufficient information, embankment foundation soils were assumed to be normally consolidated, the settlements for each embankment was estimated either by back calculating UDOT settlement data or by making assumptions of design parameters such as the vertical and horizontal coefficients of consolidation, depths to groundwater table, thickness of compressible clay layers etc.

SETTLEMENT / STABILITY

UDOT initiated a study of bridge embankment settlement estimates including a very limited study of time rate of settlement under the proposed embankment heights. Data from two sites were evaluated and used to perform a detailed analysis of settlement at the two sites. Based on which, and generalized subsurface characteristics at the other 52 bridge sites, settlements were estimated (partial list of which is presented in Table 1). The assumptions made were a) applied loads are perfectly flexible, b) foundation is semi-infinite elastic isotropic solid and c) bearing capacity failure does not occur. Ultimate settlement was calculated at the center of the new embankment using Boussinesq influence value plots for vertical stress under an embankment of infinite length.

As evident from the results presented in Table 2, the settlement time for some of the embankments was very long and therefore need to be accelerated using surcharge and wick drains to meet the schedule. UDOT initiated a study to compute time-settlement estimates based on various wick drain spacing. This was accomplished by a) determining the vertical coefficient of consolidation (C_v) from time consolidation test data and from embankment as-constructed time/settlement data (where sand drains were not used); b) determining the horizontal coefficient of consolidation (C_h) from embankment as-constructed time/settlement data (where sand drains were used); c) determining the horizontal coefficient of consolidation (C_h) from CPT pore pressure dissipation data. Computation of time-settlement estimates for various wick drain spacings were based on Equation 1:

$$t = \frac{D^2}{8xC_h}\left[\frac{\ln(D/d)}{1-(d/D)^2} - \frac{3-(d/D)^2}{4}\right]\ln\frac{1}{1-\bar{u}}$$

where t = time for consolidation
C_h = coefficient of consolidation for horizontal flow
d = equivalent diameter of wick drain
D = radius of influence of wick drains
$\bar{u}$ = average degree of consolidation

Based on the information obtained, an optimum wick drain spacing of 1.5m and depth of 21.5m were recommended.

SEISMIC CRITERIA

The Salt Lake valley is located near the eastern margin of the Basin and Range physiographic province. The valley is bordered on the east by the Wasatch Range, on the west by the Oquirrh Mountains, on the south by the Traverse Mountains and on the north by the Great Salt Lake. The valley is situated in the central portion of the intermountain seismic belt (ISB).

Major normal faults in the Salt Lake Valley region include the Wasatch fault, West Valley fault zone (comprised of Taylorsville and Granger faults), Northern Oquirrh fault and East Great Salt Lake fault zone. The Wasatch fault zone to the east of the corridor is comprised of multiple segments. The Salt Lake Valley is located adjacent to the central and most active portion of the Wasatch fault zone, namely the Salt Lake segment.

Due to setting of the I-15 Corridor in the Salt Lake valley and recent attention to the vulnerability of transportation structures to the effects of ground shaking and liquefaction caused by earthquakes, the Geotechnical Review Committee created an independent Seismic Advisory Committee to assist UDOT in establishing site specific design spectra for the project and procedures for evalauating the liquefaction potential. The committee was made up of Loren Anderson (USU), Les Youd (BYU), Jim Gates (CalTrans), Walter Arabaz (U of U), Jim Pechman (U of U), Gary Christensen (USGS), Joh Bischoff (UDOT), C.B. Grouse (Dames & Moore, Inc.).

Detailed seismic hazard evaluation was conducted by Dames & Moore as a subconsultant to PB, under the direction of the Seismic Advisory Committee. The evaluation included a) literature review; b) subsurface exploration including five borings to depths ranging from 200 to 300 feet to perform downhole geophysical analysis; c) seismic source characterization; d) ground-motion attenuation; e) seismic hazard analysis logic test model; f) probalistic and deterministic seismic hazard analysis; g) site response analyses; h) develop design spectra and i) develop guidelines for liquefaction evaluation.

Five seismic sources were identified (and the logic tree approach was used to specify parameters with magnitudes ranging from 6.25 to 7.5) for the study. The Boor et al. attenuation equations and the Dames & Moore attenuation equations were used in the probabilistic seismic hazard analysis (PSHA) for nine sites along the I-15 Corridor. The deterministic seismic hazard analysis (DSHA) was done for three of the nine sites. The analytical model used for the PSHA was based on models originally developed by Cornell (1968) and Kiureghidh and Aug (1978). These models assumed the occurrence of earthquakes was completely random. Based on the analysis, it was concluded that the site amplification along the I-15 Corridor was more appropriately estimated using representative emphirical attenuation equations than by conducting one-dimensional site response analysis.

The approach to establish the design spectra consisted of two steps. First, generic design spectra were constructed from the uniform probability, best-estimate mean spectra. Final design spectra were then constructed from published information (Somerville et al. 1995) on these near field directivity effects. Final design spectra (applicable to both horizontal components) are given by the following equations.

Northern Corridor Portion - Stiff Soil (Type 1 Design Spectrum)

10% in 50 yr:
$$\begin{aligned} PSA &= 0.27, & T &< 0.04 \text{ sec} \\ &= 1.35\ T^{0.50}, & 0.04 \leq T &< 0.18 \\ &= 0.576, & 0.18 \leq T &< 0.50 \\ &= 0.309\ T^{0.90}, & 0.50 \leq T &< 4.00 \end{aligned}$$

10% in 100 yr:
$$\begin{aligned} PSA &= 0.45, & T &< 0.04 \text{ sec} \\ &= 2.25\ T^{0.50}, & 0.04 \leq T &< 0.18 \\ &= 0.96, & 0.18 \leq T &< 0.61 \\ &= 0.615\ T^{0.90}, & 0.61 \leq T &< 4.00 \end{aligned}$$

10% in 250 yr:
$$\begin{aligned} PSA &= 0.72, & T &< 0.04 \text{ sec} \\ &= 3.60\ T^{0.50}, & 0.04 \leq T &< 0.20 \\ &= 1.60, & 0.20 \leq T &< 0.67 \\ &= 1.116\ T^{0.90}, & 0.67 \leq T &< 4.00 \end{aligned}$$

Northern Corridor Portion - Soft Soil (Type 2 Design Spectrum)

10% in 50 yr:
$$\begin{aligned} PSA &= 0.288, & T &< 0.04 \text{ sec} \\ &= 1.246\ T^{0.455}, & 0.04 \leq T &< 0.20 \\ &= 0.60, & 0.20 \leq T &< 0.56 \\ &= 0.356\ T^{0.90}, & 0.56 \leq T &< 4.00 \end{aligned}$$

<u>10% in 100 yr:</u>

$$\begin{aligned} PSA &= 0.48 &,\quad & T < 0.04 \text{ sec} \\ &= 2.076\ T^{0.455} &,\quad & 0.04 \le T < 0.20 \\ &= 1.00 &,\quad & 0.20 \le T < 0.61 \\ &= 0.641\ T^{0.90} &,\quad & 0.61 \le T < 4.00 \end{aligned}$$

<u>10% in 250 yr:</u>

$$\begin{aligned} PSA &= 0.768 &,\quad & T < 0.04 \text{ sec} \\ &= 3.322\ T^{0.455} &,\quad & 0.04 \le T < 0.22 \\ &= 1.667 &,\quad & 0.22 \le T < 0.72 \\ &= 1.240\ T^{0.90} &,\quad & 0.72 \le T < 4.00 \end{aligned}$$

<u>Southern Corridor Portion - Stiff Soil (Type 3 Design Spectrum)</u>

<u>10% in 50 yr:</u>

$$\begin{aligned} PSA &= 0.24 &,\quad & T < 0.04 \text{ sec} \\ &= 1.28\ T^{0.52} &,\quad & 0.04 \le T < 0.18 \\ &= 0.525 &,\quad & 0.18 \le T < 0.50 \\ &= 0.281\ T^{0.90} &,\quad & 0.50 \le T < 4.00 \end{aligned}$$

<u>10% in 100 yr:</u>

$$\begin{aligned} PSA &= 0.40 &,\quad & T < 0.04 \text{ sec} \\ &= 2.13\ T^{0.52} &,\quad & 0.04 \le T < 0.18 \\ &= 0.875 &,\quad & 0.18 \le T < 0.56 \\ &= 0.519\ T^{0.90} &,\quad & 0.56 \le T < 4.00 \end{aligned}$$

<u>10% in 250 yr:</u>

$$\begin{aligned} PSA &= 0.64 &,\quad & T < 0.04 \text{ sec} \\ &= 3.41\ T^{0.52} &,\quad & 0.04 \le T < 0.18 \\ &= 1.40 &,\quad & 0.18 \le T < 0.61 \\ &= 0.897\ T^{0.90} &,\quad & 0.61 \le T < 4.00 \end{aligned}$$

The final design spectra were compared with the equivalent AASHTO spectra and spectra currently used by UDOT. The AASHTO spectra correspond to ground motion with a 10% probability of being exceeded in 50 years, whereas the UDOT spectra, which have shapes identical to those of AASHTO, correspond to a 10% probability of being exceeded in 250 years. For the Salt Lake City area, the AASHTO spectra are normalized to a bedrock ground acceleration (4) of 0.29g whereas the UDOT spectra are normalized at 4=0.61g. The final design spectra recommended in this section pertain to sites where liquefaction is unlikely.

LIQUEFACTION HAZARD EVALUATION

Liquefaction potential maps developed by Anderson et. al in 1985 indicated that the entire I-15 corridor from 13800 South to 600 North is classified as having a moderate to high liquefaction potential, and specifically, the high potential exists from 50th South to 600 North. Therefore it was considered prudent to evaluate the liquefaction potential on a site specific basis along the corridor. In order to be able to accomplish this in a consistent manner, guidelines for evaluating using CPT data or SPT blowcounts was developed and established. In addition to identifying the potential, guidelines for assessing the liquefaction problem, namely lateral spreading, based on Bartlett and Youd (1995) was also established.

STRENGTH GAIN DUE TO LONG TERM EMBANKMENT LOADING

In addition to the above studies, a study was conducted to evaluate the extent/magnitude of strength gain of soft subgrade soils due to long term loading of I-15 embankments. Three locations along the corridor with relatively high (8-9 meters) embankments were selected for the study. At each location three CPT soundings were conducted, one away from the anticipated zone of influence of the embankment (free field site), one near the "toe" of the embankment and one through the "top" of the embankment into the soils under the full influence of the loading.

A general formula for deriving soil strength (S_u) from CPT data is given as:

$$S_u = (q_c - \sigma_{vo}) / N_k$$

where : q_c = tip resistance (given as Q_t in the ConeTec logs),
σ_{vo}= total overburden stress, and
N_k = cone factor constant.

Cone-tip resistance in kPa was recorded in 0.05 m increments during logging and was retrieved from an ASCII data file. Total overburden stress was calculated for each 0.05 m depth reading using a typical soil unit weight (wet) of 18.1 kN/m^3. The cone factor correction "N_k" can range from between about 5 (very soft soils) and 25 (heavily over-consolidated or cemented soils). However, for most normally consolidated to lightly over-consolidated soils, the value of N_k typically ranges between 10 and 15. ConeTec technical personnel indicated that it is rare for N_k value to fall outside the 10 to 15 range. Given this information, N_k of 12.5 was used in the CPT-based soil strength calculations. Soil strengths for each CPT tip resistance measurement were calculated by using the above equation.

Data from boring logs and laboratory tests conducted for the 600 South and 1300 South Section geotechnical investigations were reviewed and grouped, based on position relative to the embankment (Table 3). Unconfined shear strength tests results were tabulated for soil samples from seven borings located on the top of the embankment and samples from thirty borings located near the embankment toe. No borings were available to represent a free field site. For consistency, only tests conducted on low-plasticity clays (Unified Soil Classification System designation "CL") were included in this comparison. Average soil strengths were calculated for samples from similar depths and also over the entire hole.

The results of this analysis indicate, based on comparisons of test strengths averaged by hole, that the soil samples collected from borings located in areas under the influence of the embankment have soil strengths about 20% higher than samples from borings near the embankment toe.

The results of both CPT-derived and laboratory-based strength analysis suggest that soils under the full influence of the embankment have higher average strengths than soils near the embankment toe. Lab-based data indicate relative strength increases in "top" versus "toe" soils in the range of 20%, while the CPT-derived data suggest over 40% increase. The CPT data also suggest some strength increases in embankment toe soils over free field soils, however this trend has not been confirmed with similar lab-based results.

ASSOCIATED STUDIES

Pile load testing was also carried out to a) determine lateral capacities of a group of piles in soft clay; and b) correlate pile capacities using CPT data.

UDOT, in addition to the above analysis/studies, developed a) I-15 Corridor Subsurface Exploration and Laboratory Testing Guidelines; b) I-15 Corridor Soil Classification Field Manual; and c) I-15 Corridor - Geotechnical Design Guidance Manual for the project. The purpose of developing and establishing these guidelines was to ensure consistency among the various geotechnical consulting firms in the valley.

SUMMARY AND CONCLUSIONS

UDOT had the insight to determine that in order to get this largest public undertaking done on time and on budget, that it had to provide as much information about the subsurface conditions as possible. Therefore, in addition to the information presented here, UDOT conducted extensive investigations (over 1000 borings/CPT soundings) through various subconsultants.

The Historical Geotechnical Data Report serves as a baseline geotechnical report summarizing not only all previous exploratory holes, test pits on baseline maps and associated laboratory testing data, but also foundation performance data included fill settlement records, pile load tests, sand drain locations on a baseline map. These valuable data were originated from numerous independent reports prepared during design and construction of the existing I-15 corridor in early 1960s and could have easily been overlooked or lost should they not be compiled into one report.

Based upon actual settlement observation data from instrumented embankments constructed in 1960s, it can be concluded that magnitudes of settlement predicted by one-dimensional consolidation theory are comparable with the measured ones for the soft lacrustrine deposits of the Salt Lake City Valley.

Staged construction of embankments over the soft lacrustrine clays is considered necessary to prevent overall bearing capacity failure. This had been demonstrated to be the case, by the historical data retrieved from reports prepared for the original construction.

The crux of the strip drain analysis is the determination of the horizontal coefficient of consolidation, C_h. Based upon the Stage 1 study and the historical settlement data, C_h varied from 0.1 to 1.0 cm^2/min. The high C_h value probably represented seams of silt and sandy silt in the soft lacrustrine clays.

For normally consolidated to lightly over consolidated clays in the Salt Lake Valley, the cone factor constant, N_k may vary from 10 to 15. N_k value of 12.5 was used in the CPT-based soil strength calculations.

Design spectra for the three soil types were generated for 10% probability of exceedence in both 50 years 250 years. The spectra were found to be lower than the AASHTO for the first case and higher than the AASHTO for the second case at lower oscillator periods.

The depth of knowledge and experience of the people involved enabled the identification of the areas of concern and approach to take. The project has been bid and the contract has been awarded. The feedback to UDOT has been very positive in terms of the information provided to the proposers for bidding purpose.

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Fig. 1 Site Location

TABLE 1 - SETTLEMENT ESTIMATES

No.	Bridge Site	Station	C_c	Settlement (m)
1	I-15 over 9000 South	13+240	.028	0.2
2	I-15 over 7200 South	16+950	0.28	0.85
3	SR201 EB to I-15 NB over SPRR	------	0.26	1.76
4	600 South Viaduct over 400 West & UPRR	------	0.28	1.17
5	I-80 EB to I-15 NB over I-15	------	0.31	1.67

TABLE 2 I-15 CORRIDOR PRELIMINARY SETTLEMENT AND STABILITY SUMMARY FOR FILL HEIGHTS $\geq$ 18 FEET

Location	Test Hole	Fill Height Feet	Surcharge Feet	Estimated Settlement Inches	Time for 90% Settl. Days	Time for 100% Settl. Days	Wick Drains Spacing Feet (c/c)	Time for Wick Drains Days	Slope Stability. 2:1 (H:V)	Factor of Safety 3:1 (H:V)	Staged Construction Yes/No
1	No.5	43	-	9	1,170	2,070	3	140	0.59	0.63	No
		-	-	-	-	-	-	-	-	1.36/1.33/1.4*	Yes
		43	20	9	-	800	-	-			
2	No.8	40	-	10	69	144	-	-	1.57/1.45/1.6*	-	Yes
3	No.6	28	-	10	232	425	4	90	1.07	1.12	No
		28	10	10	147	232	-	-			
		28	-	-	-	-	-	-	2.1/1.7**	-	Yes
4	No.8	22	-	7	69	144	-	-	1.07	1.12	No
		22	-	-	-	-	-	-	2.2/1.7**	2.2/1.8**	Yes
5	P109/No.7	23	-	7	69	144	-	-	1.18	1.26	No
6	No.4	35	-	23	168	283	3	34	1.49/1.43**	-	Yes
8	P-51,P-129	32	-	51	3,995	5,590	3	150	1.36/1.34**	-	Yes
9	No.4	42	-	26	168	283	3	34	1.26/1.36**	-	Yes
		42	20	26	-	115	-	-			
10	No.4	31	-	21	168	283	4	90	0.84	-	No
		-	-	-	-	-	-	-	1.73/1.47**	-	Yes
		31	20	21	-	95					
11	None	35	-	47	1,215	1,700	4	100	1.4/1.5**	-	Yes
12	None	32	-	44	1,215	1,700	4	100	1.48/1.43**	-	Yes
13	No.15	20	-	12	1,140	2,030	3	140	2.5/1.9**	-	Yes
		20	10	12	-	740	-	-			
14	No.14	19	-	20	1,300	1,500	4	140	2.4/1.8**		Yes
		19	10	20	-	820	-	-			
15	P-203	21	-	55	2,935	4,100	3	150	2.0/1.67**	-	Yes
16	P-198	22		10		500			1.18	1.27	No
		22		10		300					
D & RGW	D-3	18		22		630			1.34		No
		18		22		202			1.06		No
D & RGW (3500S.)	U-1	25		15-22		510	4	90	1.88/1.57*		Yes
		25		15-22		258					

* DENOTES FACTORS OF SAFETY FOR THREE - STAGE CONSTRUCTION, THE HEIGHT OF EACH STAGE IS EQUAL TO THE OVERALL FILL HEIGHT DIVIDED BY 3.
** DENOTES FACTORS OF SAFETY FOR TWO - STAGE CONSTRUCTION, THE HEIGHT OF EACH STAGE IS EQUAL TO FILL HEIGHT DIVIDED BY 2.

LOCATION

1	I-80 Westbound to I-15 Southbound Ramp over I-15 (South Abutment)
2	I-80 Westbound to SR-201 Westbound Ramp over I-15 RR Yard (East Abutment)
3	I-15 Southbound over I-80 Eastbound Ramps (South Abutment)
4	I-80 Westbount to I-15 Northbound Ramp over I-15 Northbound CD (South Abutment)
5	I-15 Southbound CD to I-15 Southbound over I-80 Easybound Ramp (South Abutment)
6	SR201 Eastbound to I-15 Northbound Ramp over 2100S. (South Abutment)
8	I-15 Southbound to SR201 Westbound Ramp over RR Yard/I-15 (West Abutment)
9	SR201 Eastbound to I-15 Northbound Ramp over RR Yard/I-15 (West Abutment)
10	SR201 Eastbound to I-15 Southbound/I-80 Eastbound Ramp over RR Yard (West Abutment)
11	I-15 Southbound to SR201 Eastbound over 900 West (East Abutment)
12	SR201 Eastbound over 900 West (East Abutment)
13	6TH South (North Abutment)
14	5TH South (North Abutment)
15	I-80 Eastbound to I-15 Northbound Ramp over I-15 (North Abutment)
16	I-80 Eastbound to I-15 Northbound Ramp over I-15 (West Abutment)
D & RGW	6400 South Interchange
D & RGW	3500 South

TABLE 3A - TIME FOR RADIAL, VERTICAL AND TOTAL CONSOLIDATION (C_h=0.18 cm^2/min)

Wick Drain Spacing (meters)	Percent Consolidation with Radial Drainage	Time for Radial Consolidation (days)	Percent Vertical Consolidation Based on Column 3.	Percent of Total Consolidation (combined radial and vertical)
0.5	80%	3.5	6.7%	81%
1	80%	20.8	16.3%	83%
1.5	80%	55.7	26.7%	85%
2	80%	110.3	37.6%	87.5%
2.5	80%	186.2	48.7%	90%
0.5	90%	5.1	8.1%	91%
1	90%	29.7	19.5%	92%
1.5	90%	79.7	32.0%	93%
2	90%	157.9	44.9%	94.4%
2.5	90%	266.4	58.0%	95.7%
0.5	95%	6.6	9.2%	95.5%
1	95%	38.7	22.3%	96.0%
1.5	95%	103.7	36.5%	96.8%
2	95%	205.4	51.1%	97.5%
2.5	95%	346.6	65.5%	98%

TABLE 3B - TIME FOR RADIAL, VERTICAL AND TOTAL CONSOLIDATION (C_h=1.05 cm^2/min)

Strip Drain Spacing (meters)	Percent Consolidation with Radial Drainage	Time for Radial Consolidation (days)	Percent Vertical Consolidation Based on Column 3.	Percent of Total Consolidation (combined radial and vertical)
1	80%	3.56	6.67%	81%
2	80%	18.9	15.6%	83%
3	80%	48.7	25.0%	85%
4	80%	94.5	34.7%	87%
5	80%	157	44.8%	89%
1	90%	5.1	8.01%	91%
2	90%	27.1	18.6%	92%
3	90%	69.7	29.9%	93%
4	90%	135.1	41.6%	94%
5	90%	224.7	53.4%	95%
1	95%	6.6	9.2%	95.5%
2	95%	35.2	21.3%	96.0%
3	95%	90.7	34.0%	96.7%
4	95%	175.8	47.4%	97.4%
5	95%	292.3	60.6%	98%